EMERGENCY SPILLWAY PERFORMANCE, UPPER RED ROCK CREEK WATERSHED, OKLAHOMA

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EMERGENCY SPILLWAY PERFORMANCE, UPPER RED ROCK CREEK WATERSHED, OKLAHOMA

By W. O. Ree[1]

ABSTRACT

A storm on October 10–11, 1973, near Enid, Okla., released up to 19 inches of rain over the upper end of Upper Red Rock Creek watershed. Four floodwater-retarding reservoirs in this area, built under the Public Law 566 program administered by the Soil Conservation Service, received sufficient flow to cause outflow in the emergency spillways. A field survey immediately after the flood determined high-water elevations in the reservoirs and assessed the condition of the emergency spillways after the flood. Subsequent calculations were made to estimate the peak flow rates in the spillways and the related flow velocities. The spillways performed well during this flood event; however, they were not put to the ultimate test of conveying the flow rate for which they had been designed.

INTRODUCTION

Emergency spillways for upstream floodwater-retarding dams are ordinarily earth spillways excavated into the existing grade at the site, then backfilled with topsoil and planted to grass for surface protection. Emergency spillways are intended to be breachproof, but criteria to achieve this degree of stability are not available. Establishing design criteria for breachproof emergency spillways will be difficult because several site characteristics must be considered, including the erodibility of the subsoil and the presence of rock in the layers beneath the surface. The quality of grass lining is not included among the mentioned characteristics because the conservative assumption for design is that the grass might not be there when the design flood occurs. Since drought, overgrazing, burning, or vehicle traffic can weaken or destroy the grass cover,

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the stability of the spillway is judged by the mass of soil and rock material which must be removed by the flow before breaching occurs.

Experimentation on full-size structures to determine the mass of material needed to assure a safe spillway is beyond the capability of existing hydraulic laboratories. Furthermore, testing full-size spillways would not be practical because of the many different soil and rock profiles which would have to be tested. An alternative to experimentation is to observe and measure spillways that have conveyed large flows and to deduce design criteria from these observations. However, the infrequency of large flows makes this approach slow to yield results. It appears, therefore, that a reasonable approach would be to combine experimentation and field observation.

A mathematical model of the breaching process could be formulated which would include the hydrograph, the spillway width, the difference in elevation between the spillway crest and the stable downstream grade, and the erodibility characteristics of the soil and rock mass in the spillway profile. The erodibility characteristics of the soil and rocks could be determined by laboratory experiments which would provide numerical data for quantifying the model. The model in turn should be tested by application to known events; data on spillway performance during floods would be required for testing. This is a report of the data gathered on the performance of three spillways during an infrequent flow event.

At 4:00 p.m. on October 10, 1973, a rainstorm began at Enid, Okla.; by 5:00 a.m. the next morning 15.68 inches of rain had fallen. A National Weather Service bucket survey disclosed a maximum of 20 inches of rain in the center of the city, about 2 miles from the rain gage. Thirteen miles northeast of Enid and 3 miles southwest of Hunter, Okla.,19 inches of rain fell. This point is in the western part of the Upper Red Rock Creek watershed, Oklahoma, the location of a Public Law 566 project of the Soil Conservation Service.

Four of the nearby upstream floodwater-retarding reservoirs in the watershed received sufficient runoff to cause flow in the emergency spillways. These sites and the isohyetal lines for the storm are diagramed in figure 1. The emergency spillway flows aroused the interest of the staff of the Water Conservation Structures Laboratory, who visited the area and first observed the emergency spillways on October 23, 1973. The detailed surveys were made on November 1 and 2. The high-water marks in the reservoir and the crest and channel profiles of the spillways were measured, and the kind and quality of the grass cover and the condition of the spillways were noted. From these data the peak flow and the maximum velocity in each spillway during the flow were calculated.

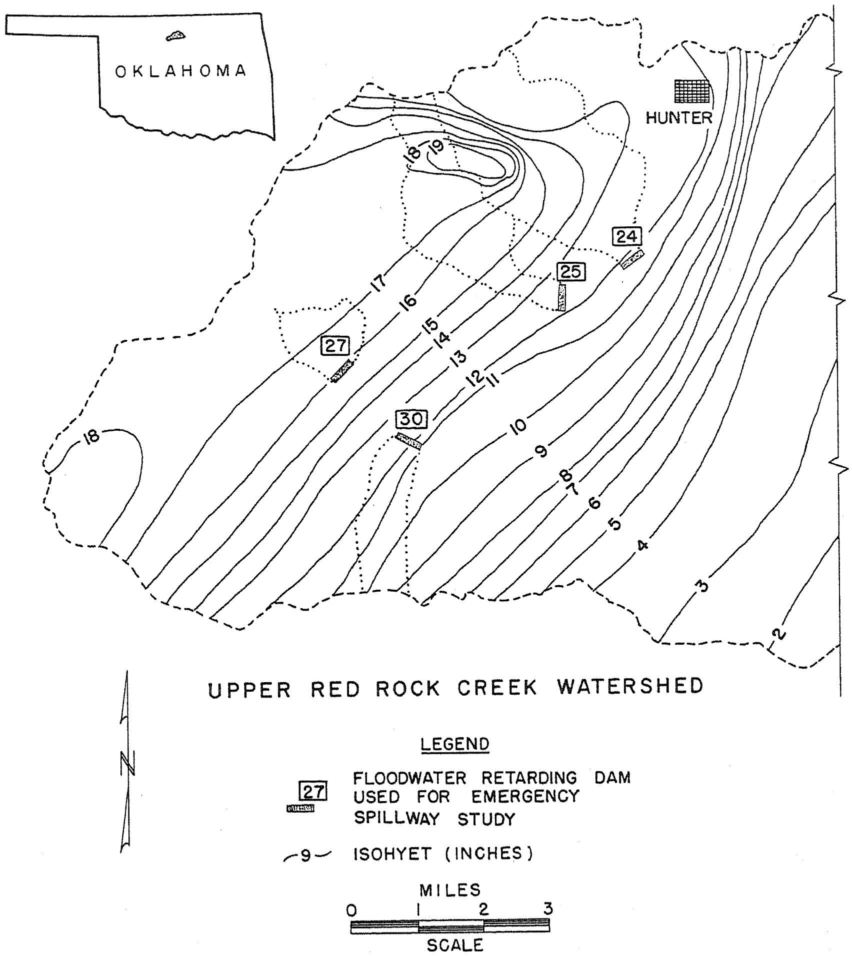

FIGURE 1.—Sites of floodwater-retarding reservoirs and precipitation for the storm of October 10–11, 1973, over the upper portion of Upper Red Rock Creek watershed. Only the sites visited are marked and identified.

Each of the three sites with a constructed emergency spillway will be discussed; the spillway characteristics are given in table 1. Site 27 has no constructed emergency spillway, but rather a grassed area to serve as an emergency spillway. No data to calculate velocities could be obtained.

SITE 24

The site 24 watershed has a flat to low, rolling topography and an area of 4,013 acres. Wheat production and cattle grazing are the main agricultural activities in the watershed. At the time of the storm some of the recently sowed wheat had sprouted. The average storm rainfall over this watershed was 14.7 inches.

TABLE 1.—*Characteristics of emergency spillways in Upper Red Rock Creek watershed, Oklahoma, during flood of October 10–11, 1973*

			Emergency spillway							
Site	Drainage area (acres)	Peak flow (ft^3/s)	Width (ft)	Slope	Soil[1]	Cover	Cover condition	Estimated retardance class	Peak flow velocity (ft/s)	Condition after flood
24	4,013	730	190	0.03319	LL=30, PI=14, D=21%.	Bermuda-grass, misc.	Overgrazed, poor.	E	6.7	Poor, some erosion over entire surface, local scour holes. Severe scour left bank at downstream end. No threat of breaching.
25	2,061	767	280	.03778	LL=42, PI=21, D=5%.	Bermuda-grass, some misc., broom-weed.	Very good, grazed.	E	5.6	Very good. One small area of light erosion in thin cover area.
30	934	50	110	.02931	LL=35, PI=17, D=50%.	Bermuda-grass.	Fair to excellent.	C	1.0	Very good. One small gully at outlet end.

[1] The original soil at the site beneath the topsoil cover, CL in unified classification. LL=liquid limit, PI=plasticity index, D=dispension percentage.

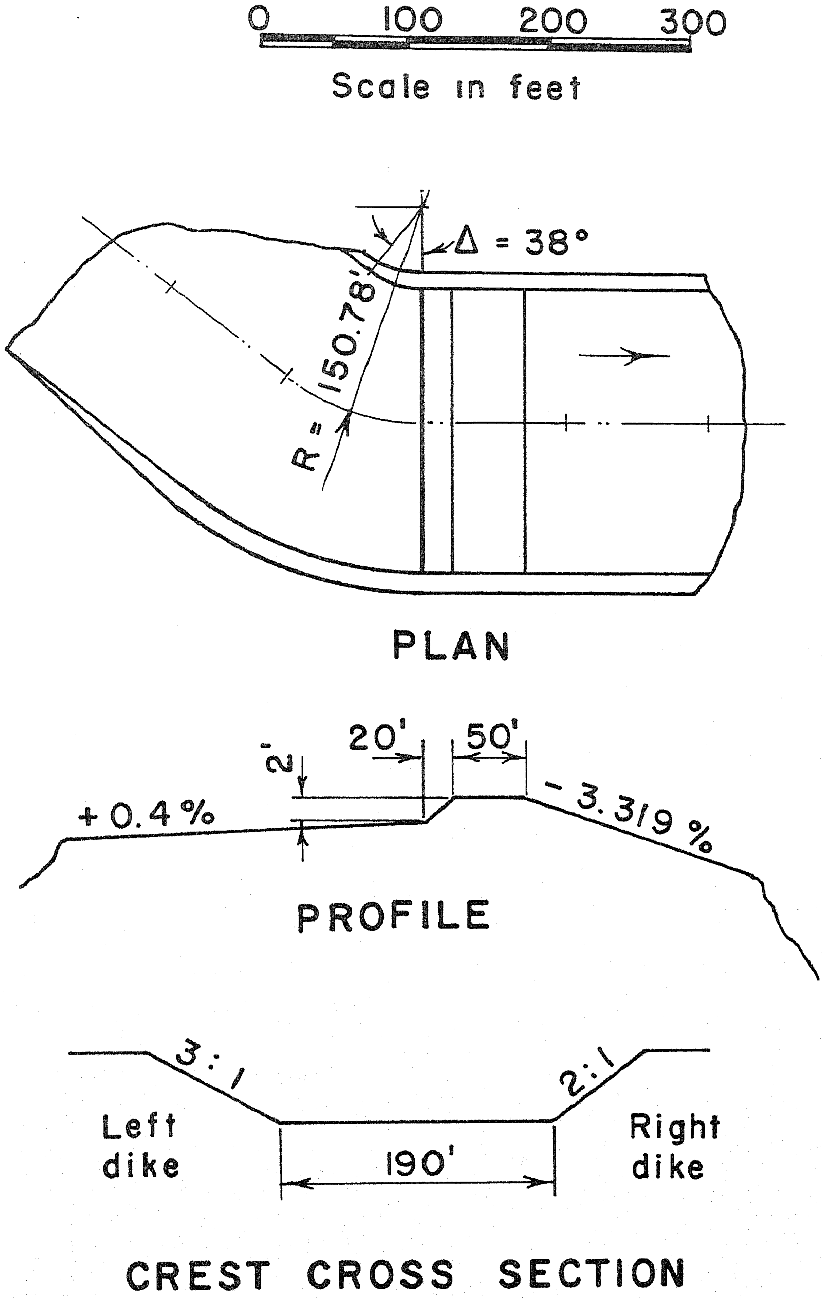

FIGURE 2.—Emergency spillway dimensions, site 24.

The emergency spillway is an earth channel around the right end of the dam (fig. 2). It was excavated in the red clay and shale of the site, topsoiled to a depth of 6 inches, and planted with ber-

mudagrass. At the time of the survey the noneroded area of the spillway had a poor stand of short, heavily grazed bermudagrass and a scattering of broomweed. The flow retardance was judged to be class E, based on the criteria in the Soil Conservation Service handbook.[2]

During the storm the water surface in the reservoir rose to a peak of 1.6 feet above the emergency spillway crest. The flow rate at this head is estimated to have been 730 cubic feet per second. (The procedure for making this flow estimate is given in appendix A.)

The normal flow velocity downstream from the spillway crest for a flow of 730 cubic feet per second was determined by the use of figures 13 and 18 in the "Handbook of Channel Design for Soil and Water Conservation" (cited in footnote 2) to be 6.7 feet per second. The procedure for determining the mean velocity is given in appendix B. A calculation of the water surface profile downstream from the crest showed that the normal flow velocity was reached within 10 feet of the crest. Therefore, for this spillway the normal flow formula provided a satisfactory estimate of the flow velocity.

What effect did this flow with a maximum mean velocity of about 6.7 feet per second have on the emergency spillway? This question is best answered by examining the photographs of the emergency spillway after the flow. Figure 3 is a view of the area

[2] Handbook of channel design for soil and water conservation. 1954. U.S. Dep. Agric., Soil Conserv. Serv. [Rep.] SCS–TP–61, 34 pp.

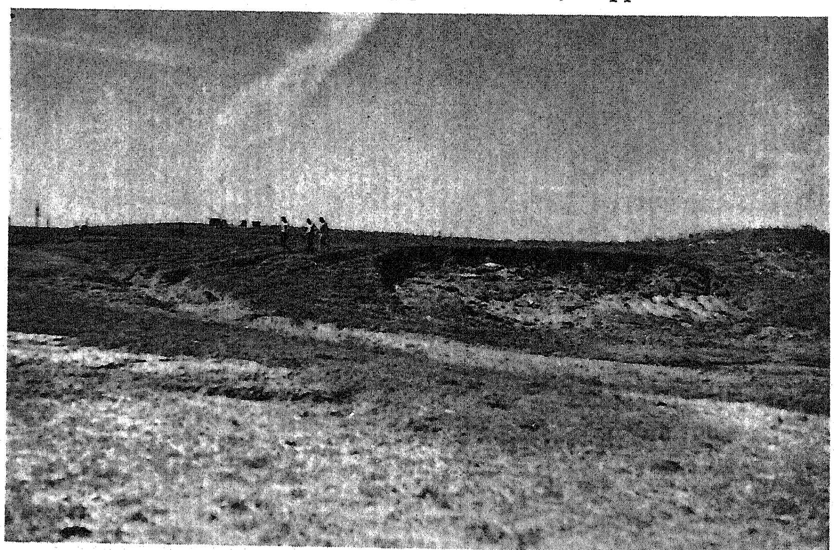

FIGURE 3.—Outlet end of site 24 emergency spillway after flood of October 10–11, 1973.

upon which the spillway discharges. The badly scoured portion to the right is on the bank of a natural channel or gully alongside the emergency spillway and was attacked by overflow from the spillway. Figure 4 shows the rills in the lower end of the spillway.

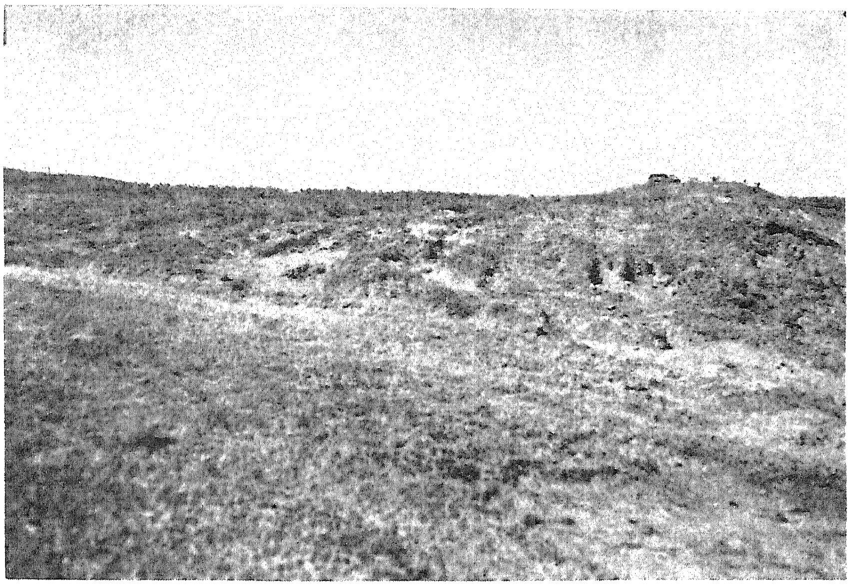

FIGURE 4.—Lower end of site 24 emergency spillway after flood of October 10–11, 1973.

FIGURE 5.—Two-foot scour to the shale bedrock 90 feet downstream from site 24 spillway crest.

Figure 5 is a closeup view of the head of one of the deeper rills. The scour here is about 2 feet deep and extends to the shale bedrock. This part of the spillway was subjected to the 6.7 feet per second flow velocity. Immediately downstream of the spillway crest is a small area scoured to a depth of 5 inches (fig. 6).

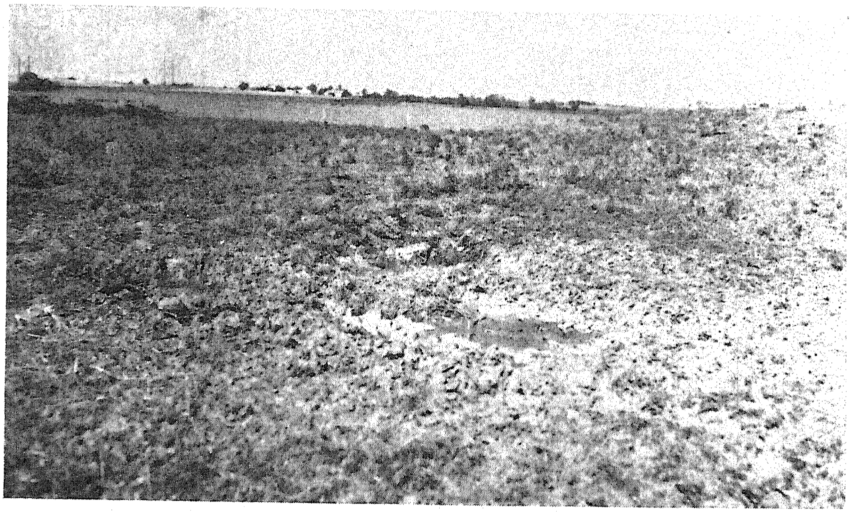

FIGURE 6.—Scoured area in site 24 spillway just downstream from spillway crest. The cover on the adjoining area is thin, poor, and short.

The site 24 spillway started to erode during a flow event which had a peak mean velocity of 6.7 feet per second. The spillway appeared to have been heavily grazed, and this may have been responsible for its condition. Cattle trails and "stomp" areas were present in the spillway at the time of inspection. Therefore, it is reasonable to assume that the eroded areas also had suffered from similar cattle activity.

SITE 25

The 2,061-acre watershed above site 25 has a relatively flat topography. It is used mainly for wheat production and for some cattle grazing. There are a few trees around the farmsteads and creek bottoms. During the storm the watershed received 16.2 inches of rain, a weighted average obtained from the isohyetal map.

The emergency spillway is at the right end of the dam. Its dimensions are shown in figure 7. A minimum of 6 inches of topsoil had been placed on the spillway surface after excavation and shaping. At the time of the flood the spillway had a very uniform, good cover of bermudagrass, with a scattering of a short, fine-leaved grass and broomweed. The spillway had been grazed, and the grass was relatively short. Its flow retardance was estimated to be class E.

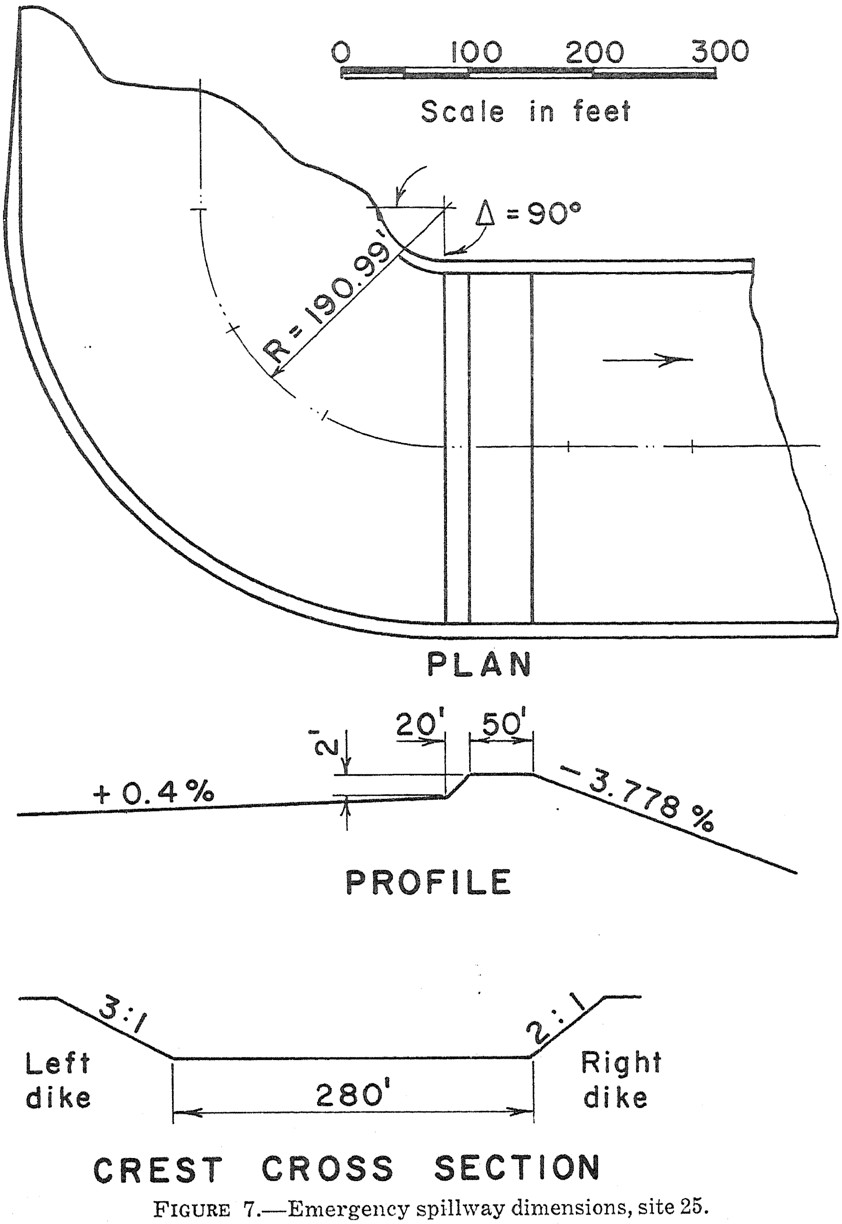

FIGURE 7.—Emergency spillway dimensions, site 25.

Runoff from the storm was sufficient to cause flow in the emergency spillway. The water surface elevation reached a maximum of 1.3 feet above the crest of the emergency spillway. The peak outflow rate for this head was estimated to be 767 cubic feet per second (by the procedure described in the presentation of the site 24 data). A water surface profile computation for a discharge of 767 cubic feet per second showed that the slope of the spillway was

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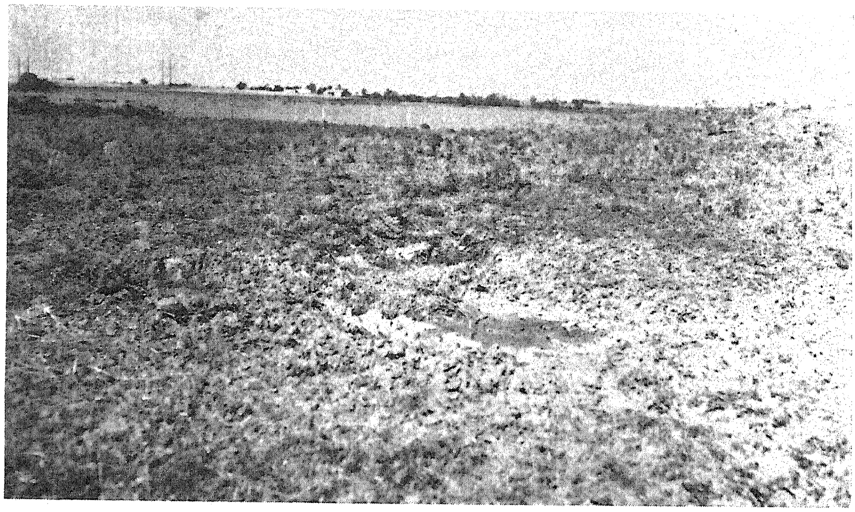

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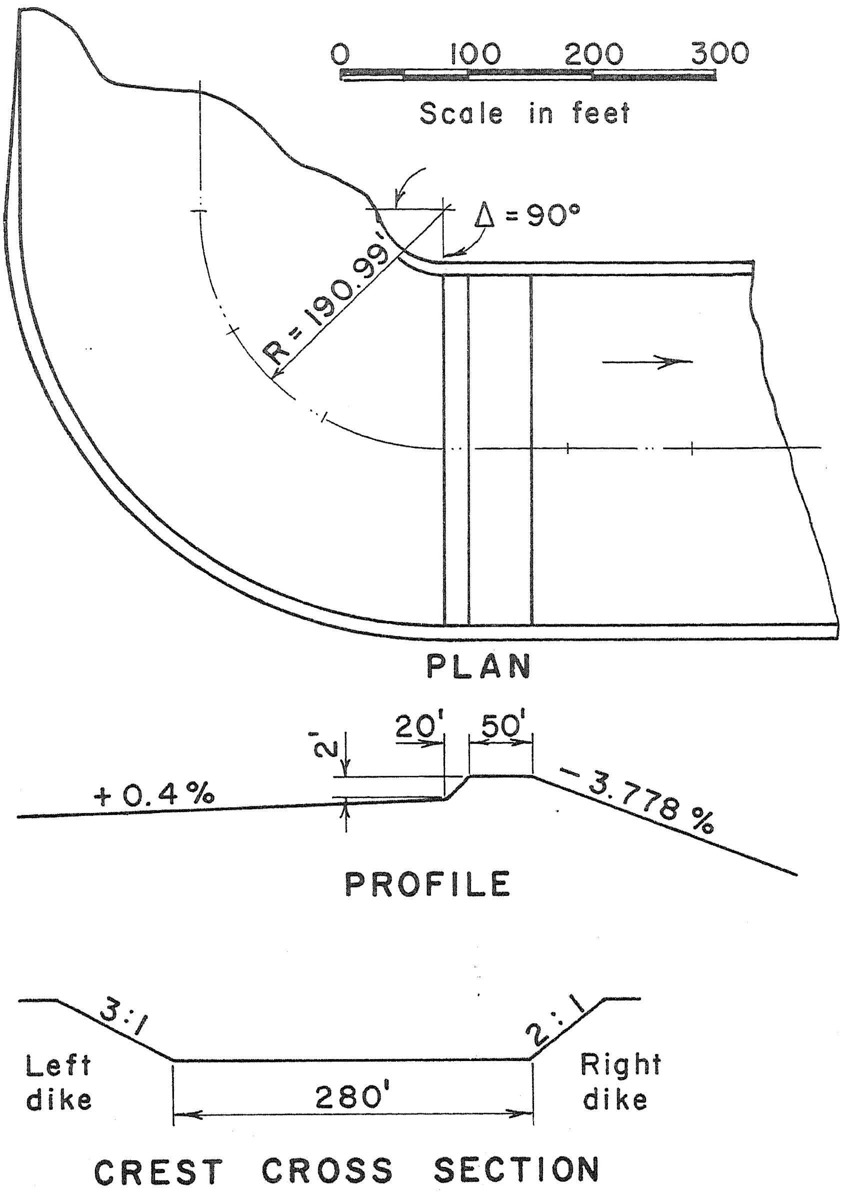
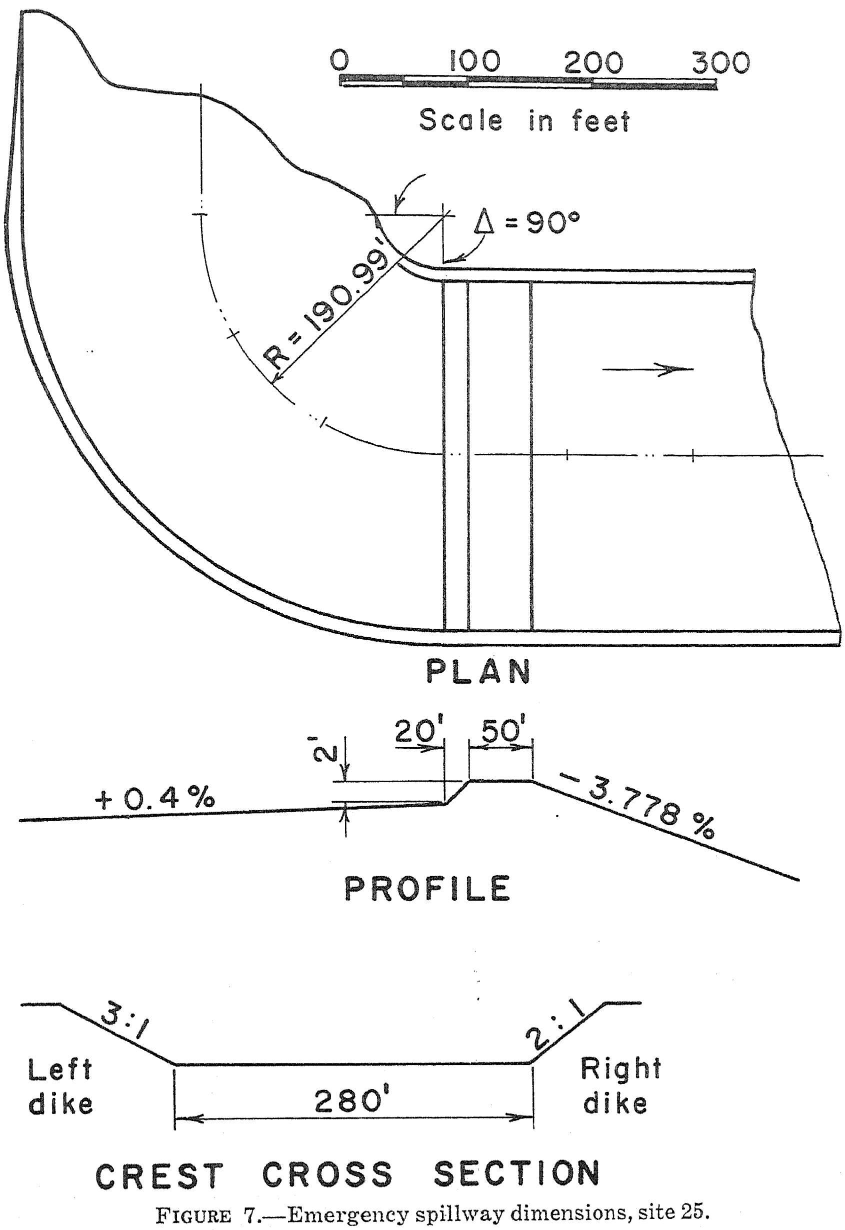


FIGURE 7.—Emergency spillway dimensions, site 25.

Runoff from the storm was sufficient to cause flow in the emergency spillway. The water surface elevation reached a maximum of 1.3 feet above the crest of the emergency spillway. The peak outflow rate for this head was estimated to be 767 cubic feet per second (by the procedure described in the presentation of the site 24 data). A water surface profile computation for a discharge of 767 cubic feet per second showed that the slope of the spillway was



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slightly greater than critical, so the flow accelerated slightly after passing over the crest and reached a mean velocity of about 5.6 feet per second. During the peak flow this velocity was uniform for nearly the entire length of the downstream portion of the spillway.

The spillway was in excellent condition after the flood. Some slight erosion was evident about 20 feet downstream from the crest and about 40 feet in from the right bank (fig. 8). It is not known whether this eroded area was present before the flood or whether there had been a weak spot in the cover here. An overall view of the spillway is shown in figure 9.

The excellent condition of the spillway should be expected because the flow velocity of 5.6 feet per second is below a suggested permissible velocity of 8 feet per second for a bermudagrass cover on clay soil and slopes of less than 5 percent, as determined from the "Handbook of Channel Design for Soil and Water Conservation" (cited in footnote 2). Uniformity of cross section and cover and moderate grazing can also be given some credit for the fine performance of the spillway.

SITE 30

The watershed above site 30 is also flat to gently rolling wheatland and range. Its 934 acres received 10.3 inches of rain during the storm. The emergency spillway operated during this event with a maximum head of 1.06 feet and an estimated peak flow rate of 50 cubic feet per second.

The emergency spillway is at the left end of the dam. Its dimensions are shown in figure 10. This spillway has two unusual characteristics. The crest is constructed in fill, probably not more than a foot or two deep, and there is a curve in the downstream reach of the spillway. Topsoil had been placed in the spillway to a depth of 6 inches and bermudagrass was established as a cover. At the time of the flood there was an excellent stand of bermudagrass on the crest and the upper part of the spillway. Toward the lower end of the downstream reach the grass cover was fair to good. The spillway had not been grazed prior to the flood. The overall flow retardance for the cover was estimated to be class C.

The flow velocity in the upper part of the spillway was estimated to have been 1.0 feet per second at the peak of the flood. This low velocity did not erode the spillway; the grass has hardly been disturbed. Figure 11 shows the crest of the spillway after the flow.

In the lower part of the downstream reach the flow tended to concentrate in the center of the spillway, where it is no longer flat bottomed, but is lower in the center so it can merge gradually into

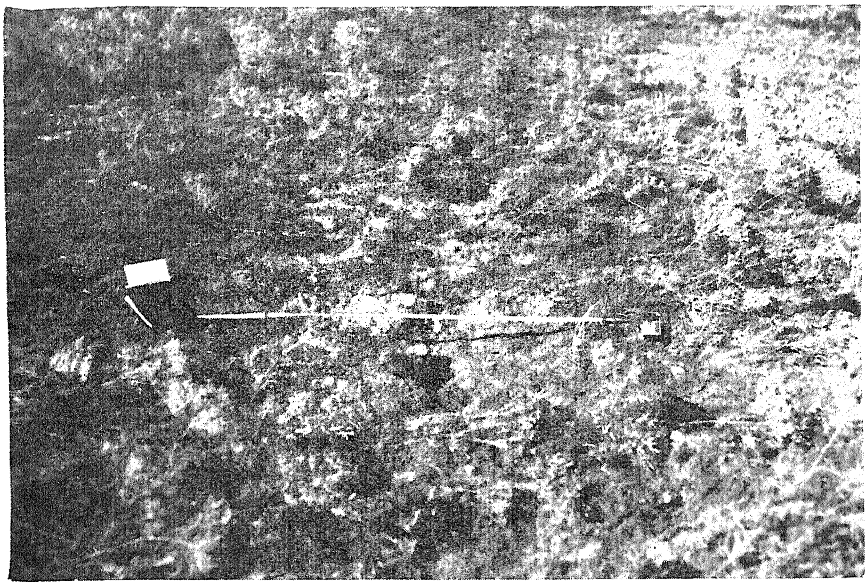

FIGURE 8.—Closeup of most eroded portion of site 25 emergency spillway after flow event of October 10–11, 1973. This point in the spillway is located approximately 20 feet downstream from the crest and about 40 feet in from the right bank. There appears to be more broomweed here, which may be a weaker cover in this area. Its appearance before the flood is not known.

FIGURE 9.—Overall view of site 25 emergency spillway after flow event of October 10–11, 1973. This spillway is in excellent condition. The cover is closely grazed bermudagrass, with a scattering of broomweed and some small, fine-leaved grass.

the topography of the site. A small gully has formed here (fig. 12). (It may have been there before, only to be enlarged by the flood.)

Three general observations can be made about this spillway.

First, constructing a spillway in fill is not an acceptable practice and is resorted to only when it is the best alternative solution to a vexing location problem. I have seen only one other spillway with a portion constructed in fill, and this portion failed during a flood event by gullying to a depth of 5 feet. This experience supports the present practice of avoiding fill in the spillway. However, there may be some advantages to using fill in a spillway. The filled area

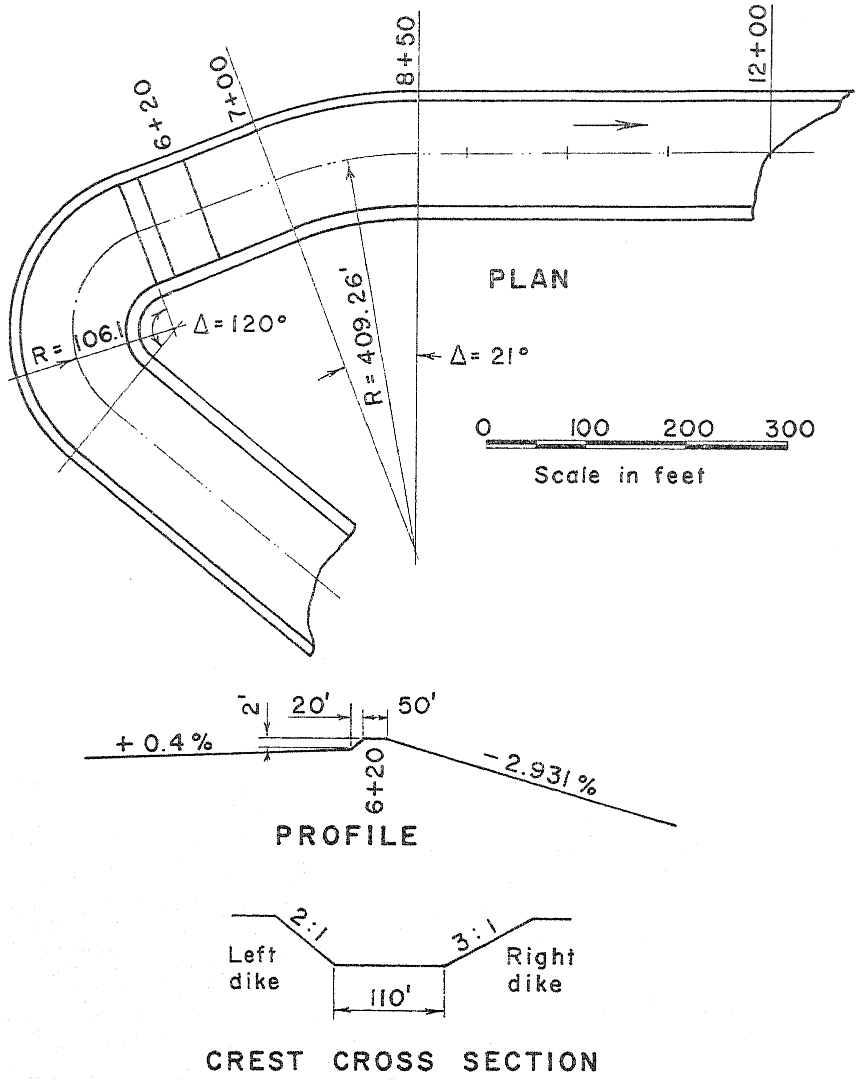

FIGURE 10.—Emergency spillway dimensions, site 30.

FIGURE 11.—Crest of emergency spillway, site 30. The cover is an excellent stand of bermudagrass.

FIGURE 12.—Small gully at outlet end of site 30 emergency spillway. Concentration of flow and the poor cover may have caused this minor failure.

in the site 30 spillway had the best stand of bermudagrass; in fact, it was luxuriant. Was the fill responsible? The question is left for future consideration.

Second, at the outlet the spillway was gradually deepened at the centerline so it could join the existing draw in a smooth transition. The flow is concentrated at the deepest point of the spillway,

where the velocity is highest and where the gully starts. (It could eventually reach the center.) A level sill and a low drop structure at the terminal end of the spillway would eliminate the need for the deepening that threatens the stability of the spillway. This would be a practical solution only if the grass could be relied upon to provide protection against erosion in the channel upstream from the terminal structure. If the grass lining is not dependable, there would be no point in using a terminal structure because the spillway could still fail from erosion above the structure. Again, the exploration of this idea is left for future consideration.

Third, every effort should be made to avoid placing curves or changes in alinement in steep channels because of the hydraulic disturbances they cause. However, in this case, a curve was the best solution to the location problem. Since the flow was not large enough to be greatly disturbed by the curve, a test of the effect of the curve on the stability of the spillway channel was not obtained during this event.

SITE 27

The 627-acre watershed above site 27 received 16.7 inches of rain. Excess runoff (1.6 feet deep) flowed over the emergency spillway crest during the storm and discharged onto a well-grassed hillside to the left of the dam. No channel-type spillway was needed here because the hillside area was adequate. This is an excellent emergency spillway arrangement, especially where the water can spread rather than converge as it flows down the hill. However, without defined flow boundaries the flow velocity cannot be estimated.

APPENDIX A.—PROCEDURE FOR COMPUTING HEAD-DISCHARGE RELATIONSHIP FOR EMERGENCY SPILLWAYS

The cross section of the spillway opening alone does not control the discharge rate because of friction loss in the approach channel and over the crest, which for broad-crested, vegetation-lined spillways can be significant. Therefore, the estimate of the discharge rate requires the calculation of water surface profiles from the control point in the spillway upstream into the reservoir to determine the relationship between the reservoir water level and the discharge rate. The procedure for making this determination will be illustrated by application to the site 24 emergency spillway in the following steps:

1. Assume that the control point is the downstream edge of the spillway crest and that the flow will be at critical depth.

2. Choose a water surface elevation at the control point.
3. Calculate the critical discharge rate for this elevation by

$$Q=\sqrt{\frac{A^3 g}{T}},$$

where Q=discharge rate,
A=cross-sectional area,
T=top width,
and g =acceleration of gravity.

Curves for elevation-area and elevation-top width developed from survey measurements facilitate the determination of A and T for a selected elevation. (A note of caution: Use actual A and T values determined by the survey rather than those obtained from plans or drawings; otherwise, large errors in the estimate of Q can occur. Also, high-water elevations should be based on the same datum used for the profile and cross-section survey.) For site 24 the values for the first choice of water surface elevation are as follows:

Water surface elevation=1070.20 feet.
A=182.9 square feet.
T=190.6 feet.

$$Q=\sqrt{\frac{(182.9)^3\, 32.2}{190.6}}.$$

Q=1,017 cubic feet per second.

Critical velocity, then, is this discharge rate divided by the cross-sectional area of the flow at this point.

$$V_c=\frac{Q}{A}=5.56 \text{ feet per second.}$$

4. Determine the Manning n value for the calculated discharge. Estimate the VR value for the flow by dividing the calculated discharge rate by the bottom width of the spillway. This provides a satisfactory estimate of the VR because the cross section for emergency spillways is usually very wide and shallow. For this example the estimated VR value is 1,017/190.0=5.35. For class E retardance, according to figure 13 in the "Handbook of Channel Design for Soil and Water Conservation" (cited in footnote 2) the corresponding n value is 0.025.

5. Calculate the normal flow velocity in the channel downstream of the crest for a flow with the same cross-sectional area and top width as for the assumed control point by the Manning equation

$$V_0=\frac{1.486}{n}\left(\frac{A}{T}\right)^{2/3} S^{1/2},$$

or $$V_0=\frac{1.486}{0.025}\left(\frac{182.9}{190.6}\right)^{2/3}(0.03319)^{1/2},$$

or $$V_0=10.5 \text{ feet per second,}$$

where V_0=normal flow velocity, feet per second,
n=Manning n, determined in step 4,
S=slope, feet per foot,
and A and T are as previously defined.

Since $V_0 > V_c$, the control point for the flow is at the downstream edge of the spillway crest. Proceed to step 6 for the calculation of the water surface profile. Actually, the value of V_0 just calculated is not the normal flow velocity because the value of Manning n used in the calculation is not necessarily correct. It would likely be less because of the higher velocity. Yet it is not necessary to repeat the computation with a new value for n because the result would be an increased V_0, only reaffirming that the control point has been correctly located.

6. Starting with the water surface elevation selected in step 2, calculate a water surface profile upstream from the control point into the reservoir, using a standard step method such as presented by Chow.[3] The Manning n value will be constant for a given discharge because the VR value will change but very little from station to station.

7. The water surface profile computation was terminated 200 feet upstream from the control point because the increase in water surface elevation was but 0.005 of a foot in the last 50 feet. The elevation reached at the final station was 1,070.89, a head of 1.79 feet above the spillway crest. This head is plotted against the corresponding discharge rate in figure A–1.

8. The reservoir water surface elevation reached in the profile calculation was not the same as the peak elevation during the storm, so another trial is needed. This time a lower water surface elevation will be selected for the starting point. If the initial trial had yielded an elevation too low, the second trial would have been started at a higher elevation. In the example, the second trial started at elevation 1,070.00, and steps 1 through 7 were followed again. The result was a discharge rate of 723 cubic feet per second and a head of 1.50 feet. This point is also plotted in figure A–1.

9. Enter figure A–1 with the storm peak head of 1.6 feet and read a discharge rate of 730 cubic feet per second.

10. If the test in step 5 had yielded a normal velocity lower than critical, then the control would not have been the cross section at the downstream edge of the spillway crest, but instead would

[3] Chow, Ven Te. 1959. Open channel hydraulics. 680 pp. McGraw-Hill, New York.

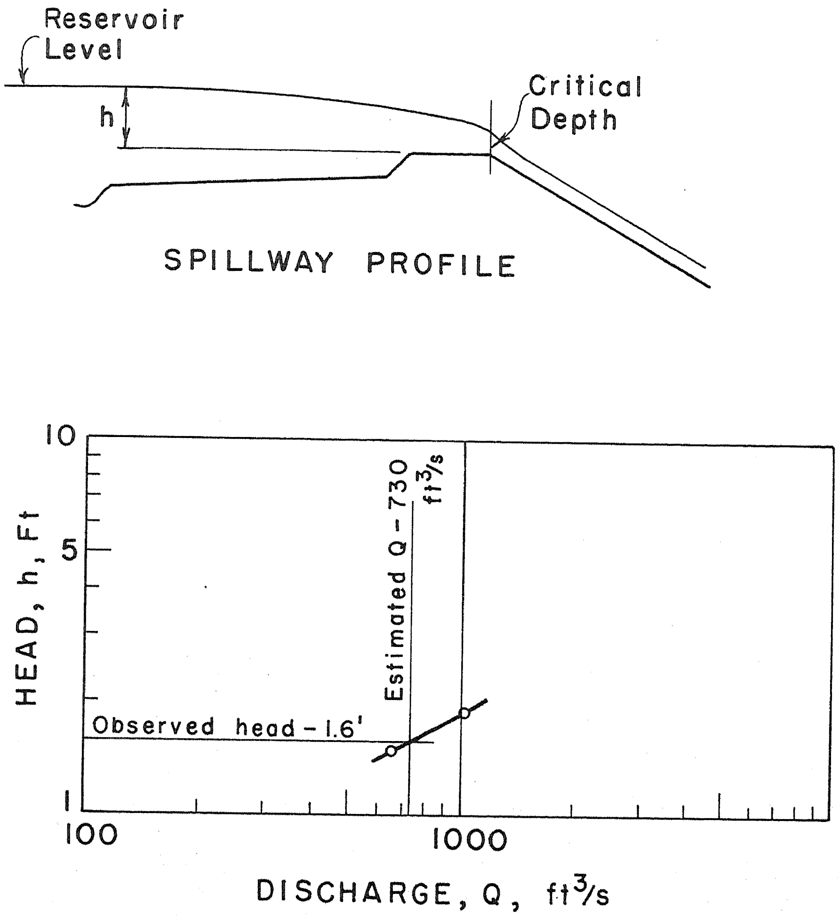

FIGURE A-1.—Rating curve for site 24 emergency spillway and estimated discharge for the flow event of October 10–11, 1973.

have been the channel below. In that event a different procedure, described in steps 11 through 17, would have been needed.

11. Start near the downsteam end of the spillway channel.

12. Select a depth and a Manning *n* value. Assume this to be a normal flow depth and calculate the normal flow velocity by the Manning equation. Check the initial *n* value for agreement with the retardance class *n* for the *VR* value obtained from the normal flow calculation. If the two *n* values are not alike, select a new value of *n* for the trial. Reiterate until agreement is reached.

13. Calculate the discharge rate as the product of normal flow velocity and area of cross section for the selected depth.

14. Calculate a water surface profile from the downstream starting point upstream, over the crest, and into the reservoir using a standard step method.

15. Repeat the process, starting with step 11, using a new depth at the beginning point.

16. Continue trying different starting depths until the desired reservoir elevation is bracketed. Two trials should generally suffice.

17. Plot the head-discharge values and determine the discharge rate needed by entering the diagram with the storm maximum head as before.

APPENDIX B.—PROCEDURE FOR ESTIMATING MEAN VELOCITY IN A GRASS-LINED SPILLWAY FOR A GIVEN DISCHARGE RATE

1. Estimate the VR value by dividing discharge rate by channel bottom width.

$$VR=\frac{730}{190}=3.84.$$

2. Enter figure 13 in the "Handbook of Channel Design for Soil and Water Conservation" (cited in footnote 2) with the VR value, and for retardance class E read $n=0.027$.

3. Enter figure 18 in the "Handbook of Channel Design for Soil and Water Conservation" with a slope of 3.319 percent and an n value of 0.027 to read the values for $R=0.57$ feet and $V=6.7$ feet per second.

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